APPENDIX 13-2

MECHANICAL LAYOUT ALTERNATIVES EVALUATION TECHNICAL MEMORANDUM

BLACK & VEATCH
POWELL KUGLER

TECHNICAL MEMORANDUM

South Florida Water Management District — B&V Project 141921.610
EAA Reservoir A-1 — B&V File B
Work Order 10 — First Issue: July 7, 2005
Last Updated:

Task 2.7.2 Pump Evaluation Technical Memorandum

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Task 2.7.2 Pump Evaluation Technical Memorandum

To: Distribution

From: Zan Kugler, Denton Voss

1. TECHNICAL MEMORANDUM

This Technical Memorandum (TM) is the Conceptual Alternatives Screening Process for the new Northeast Pumping Station to the EAA Reservoir (A-1) Project. The purpose of this TM is to identify, analyze, and screen alternatives including: horizontal configuration, vertical configuration, viable manufacturers, and pump capacity combinations for the pumping station. This TM has developed the conceptual design of (4) alternatives for comparison by a life cycle cost analysis (LCC).

The station's function is to pump S2/S7 basin runoff and lake releases from the North New River Canal to the reservoir. ***For the purpose of this TM, the pumping capacity was established at 3000cfs. Actual capacity may be more or less depending on other analysis being conducted in parallel with this analysis.*** The station shall have the capability of pumping water from the canal's design low water stage to the proposed reservoir's maximum stage of 20.6 NAVD.

The goal of the EAA Reservoir A-1 is to provide approximately 190,000 ac-ft of water storage on District owned property north of the newly constructed STA 3/4. To achieve this goal it is anticipated the reservoir will contain a maximum water depth of approximately 12 feet of water. This Reservoir A-1 is the first phase of an ultimate reservoir system that could store approximately 360,000 ac-ft of water over the entire 35,000 acres of District-owned land between the North New River and the Miami Canal. Operation of the EAA Reservoir A-1 will be limited to the flows that can be moved through the existing canal system. As part of this design, efforts shall be made to construct an economical pump configuration that meets the available flows now with provisions made for expansion for future flows when the canals are enlarged.

2. SUMMARY OF ALTERNATIVE ANALYSIS

The (4) alternatives reviewed include a (3) 1000 cfs vertical pump station with a similar design as G370 the newly commissioned inflow station for STA 3/4. A (3) 1000 cfs horizontal pump station, a (4) 750 cfs vertical pump station and a (5) 600 cfs vertical pump station. Each station has a unique mechanical arrangement. Table 1 summarizes the LLC analysis.

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Given the relative minor difference in the present value totals for the alternatives and the margin of error in the estimating of not only the construction costs but also the potential pump performance at this early stage of the design, the LCC analysis does not provide a conclusive result as to the optimum station alternative. But there is an important finding that is evident from the analysis. For this station, which could see many more hours of continuous duty than the typical District pumping station, energy costs are extremely important. This is especially true considering the cost of fuel is escalating at a rate beyond that of the 3.5% inflation rate used in the analysis. Therefore the station design needs to incorporate every efficiency measure that is available to minimize system losses and reduce fuel consumption regardless of the initial cost. This will result in the optimum station design from a life cycle cost perspective.

As a consequence of the uncertainty of the future operation of the station, flexibility of operation ranks next in importance to fuel economy in the station design. Therefore, at this point in time given the initial costs for all the alternatives are in essence equal, a five (5) pump station using smaller pumps is our recommendation. The actual size of the pumps will be determined after the completion of the other parallel analysis. The general arrangement section drawing entitled "Alternative 5" is a suggested arrangement that makes use of the advantages of all of the alternatives of the analysis and provides the greatest flexibility for operation of the current and future water supply system. This station design has the following advantages:

- Self-priming with no vacuum system required reducing potential complications for remote operation of station.
- The most number of pumps with the smallest capacity provides the most pumping flexibility.
- With the most number of pumps, provides least impact to station operation when a pump unit is off line.
- Low entrance losses due to bell intake.
- Lowest pipe crest elevation requiring low start-up horsepower without vacuum priming.
- Recovery of velocity head and low exit losses due to discharge tunnel.
- Steel fabricated discharge tunnel to reduce friction losses and lessen construction cost.
- Lowest operating static heads due to siphon assisted delivery.
- Low friction losses due to slower flow velocities as a result of the use of a larger impeller and slower rotative speed for the smaller pump capacity.
- Reduced height of substructure due to "through the levee" discharge arrangement.
- Control room and break rooms are at the opposite side of pump house from the engines permitting optimum viewing of operating floor equipment and engine control panels.
- Control room and break rooms are at the opposite side of pump house from the engine and exhaust system reducing noise in those areas.

- Engines are located close to the exterior wall permitting intake ventilators to be located in close proximity to engines for an optimum ventilation arrangement.
- Large operating floor area permitting ample room for equipment maintenance.
- Electric start engines eliminating need for large capacity compressed air system and potential complication for remote starting.
- Reduced cost of substructure due to rectangular intake.

3. CONCEPTUAL ALTERNATIVES SCREENING PROCESS

3.1 Introduction

The following (TM) for the new Northeast Pumping Station to the EAA Reservoir (A-1) Project develops and details the design of (4) alternatives for comparison by a life cycle cost analysis (LCC). The station site has tentatively been defined as the northeast corner of the proposed reservoir and shall connect to the North New River Canal via an approach canal that will include a highway bridge for US 27 which parallels the North New River Canal to its west. The station's function is to pump basin runoff and lake releases from the North New River Canal to the reservoir at nominal rate of 3000 cfs. The station design alternatives of this report do not include provisions for seepage pumping or the highway bridge. The station's pumps shall have the capability of lifting the water from the canal's design low water stage to the proposed reservoir's maximum stage of 20.6 NAVD. The station shall be designed in accordance with the District's criteria and standards for such pumping facilities.

3.2 Hydrologic and Hydraulic Design Criteria

Based on historical flow data and current modeling efforts the annual station pumping operation use for this analysis is as follows:

- Wet Season: 30.6 days (734.4 hrs) @ 2000 cfs with an average reservoir stage of (14.21 ft. NAVD).
- Dry Season: 35.32 days (847.7 hrs) @ 3000 cfs with an average reservoir stage of (16.12 ft. NAVD).
- An annual total of 331,165 ac-ft of water was pumped to the reservoir in accordance with this annual hypothetical operational scenario.

It should be noted that the existing station G370 could satisfy a similar operational plan without modification and still optimize the treatment capacity of the STA. Given all reservoir inflows are required to be discharged to the STA, the STA's treatment rate governs the inflow and the resulting reservoir stage schedule.

3.2.1 Protection Elevation

The operating floor elevation should limit the possibility of damage caused by flooding, to the pump equipment. The elevation is dependent on the maximum design water elevation that can be expected at the discharge pool. However, the clearances necessary for the below the base plate pump discharge and its coupling may be the critical dimension and must be coordinated with the pump manufacturer. The current reservoir embankment design has the top elevation set at 28 ft. above the top of rock or approximately elevation 34.6 ft. NAVD, or approximately 14.0 ft. above the maximum pool stage of the reservoir of 20.6 ft. NAVD as a result of possible wind set-up and wave heights. The reservoir embankment design is in the conceptual design stage and will be subject of continued study and modification. The Alternative No. 1 station of this analysis has an operating floor elevation set at 32.6 ft. NAVD. The other alternatives with a "through the levee discharge design" shall be provided with a flood wall to protect the station. On the North New River canal side a (4 ft.) minimum above the design flood canal stage was considered as the design criteria. None of the station alternatives approached this minimum elevation because of the vertical dimension requirements of the pumping equipment.

3.2.2 Approach Channel Considerations

The approach canal to the station shall intersect the North New River at 90 degrees and proceed a short distance west under a bridge crossing for US 27 and to the northeast corner of the reservoir. The flow approaching the pump intake should ideally be steady and uniformly distributed both laterally and vertically. The approach flow asymmetry and unsteady flow conditions are caused by the geometric layout of the approach channel and the intake. In practice it is not possible to completely eliminate non-uniform or unsteady flow conditions. The ideal hydraulic condition is for the approach channel to be in line with the intake centerline. The structure wingwalls should be at an angle of no more than 10 degrees from the centerline. In addition, the station approach channel will require a decrease in bottom elevation to the intake. This transition slope should not exceed 10 degrees. It should also be noted, a surface drop can occur across a partially blocked trash rack or whenever the pumps have lowered the liquid level in the sump to the point at which all pumps are about to be switched off. Therefore, the path between the sump entrance and the pump inlets must be sufficiently long for the air bubbles to rise to the surface and escape before reaching the pumps. The energy of the falling water should be dissipated sufficiently so that excessively high and irregular velocities do not occur within the sump. This can be accomplished with properly designed and placed baffle walls.

3.3 General Mechanical Arrangement Alternatives

3.3.1 Station Design Considerations and Criteria

One objective of the pump selection and the station design is to achieve the lowest total head to provide the smallest driver and therefore the lowest energy cost. However, flood control pump stations are operated relatively infrequently, maybe less than 1500 hrs. per year. Therefore energy savings is not as critical a design parameter as it would be for water supply or other applications where the pumps run continuously. However, the EAA Reservoir pump station shall have the reliability considerations afforded a pumping station but shall also be assumed to see continuous duty as a water supply station. Efficiency of its mechanical equipment as well as the station design will be an important factor in its design.

3.3.2 Design Life

There are a number of Corps references in regard pump station service life. According EM-1110-2-3104, EM-1110-2-3105, and the District's Major Pumping Station Engineering Guidelines, the design life of a pump station is 50 years. It is anticipated the mechanical equipment will required rehabilitation or replacement at least once during the 50 year service life. The engines and pumps will operate intermittently and with proper preventative maintenance the engine should have a 25 year service life. Pump equipment shall be designed and manufactured to ensure a long service life for the non-wear components.

The engineering regulation ER 1110-2-8159, "Engineering and Design - Life Cycle Design and Performance" defines the engineering policies for selection of all systems, components, and materials for civil works projects on the basis of their long term performance. This regulation requires the design engineer for civil works projects to use life cycle design as the basis for selection of all project elements such as materials, structural systems, mechanical equipment, and site appurtenances. Life cycle cost analysis is an important and increasingly standard method to compare alternatives of major hydraulic facilities during the conceptual development of the project. Therefore for the BODR project phase a life cycle cost analysis shall be provided of the proposed alternatives for evaluation and recommendation of the selected alternative.

3.3.3 Reliability and Cost Considerations

Flood protection pump stations should be considered emergency facilities. Equipment and power supply are specified and selected primarily on the basis of reliability under emergency conditions. Pump stations are one of the more vulnerable features of a flood protection project. Dependability must be a primary consideration during the design and selection process.

The designer needs to be aware of cost, but, because of the typically infrequent operation of flood control pump stations, efficiency can sometimes be sacrificed to a degree in favor of equipment with a lower first cost. However, for large stations with frequent usage, higher pump efficiencies can lower the installed horsepower requirements and reduce capital and operating costs significantly. Therefore, the engineer should consider higher efficiency equipment on a life cycle basis. Auxiliary systems should be minimized, the more things that can go wrong will. Refinements that make no realistic contribution to the usability or dependability should be avoided. The equipment selected must be rugged, reliable, and well suited for the service. The station should be of a robust design that is sized to house and support the equipment. Since the station can potentially be in service during extreme weather conditions when commercial power is not available, the pumps are engine driven and the station auxiliaries have backup power by engine driven generators.

Water supply, seepage return, or other pumping systems not associated with flood control or drainage do not have the same demand for high reliability. Their operation is not critical to the health and well being of the public, down time can be tolerated with repairs addressed in non-emergency mode. The drivers can be electric motors where power is available, power outages have less impact.

3.3.4 Number of Pumps

An initial step in the development of the station design is the determination of the suitable number of pumps to meet the flow demand. The operational model typically indicates a logical number of pumps of various rated capacities that when combined in parallel operation, satisfy the operational flow rate requirements. Traditionally, a minimum of two pumps should be provided with three pumps typically specified unless the foundation conditions warrant a larger number of units. For this pumping station, there should be a one pump redundancy in case of pump failure. Generally, the number of pumps should be kept to the minimum to reduce equipment capital cost, maintenance cost, and minimize the footprint of the intake to reduce construction costs. However, it may be more cost effective to have a greater number of smaller pumps to reduce the depth of the intake as well as reduce foundation loads. Smaller pumps also provide for more operational flexibility to address less than maximum flow operations. Considerations for selection of the number of pumps include:

- Station cost
- Station reliability/availability
- Maintenance cost
- Energy cost
- Operational demands and flexibility
- Foundation loads
- Driver horsepower
- Availability of pump models
- Intake depth

The historic hydrologic data summary presented earlier indicated the need for small incremental (500 cfs) capacity capability for the station. However, this historic data most probably will not represent the future conditions with the enlargement of the North New River canal and potential inter-basin transfer of runoff discharges. There are no operational criteria available to determine a needed incremental flow rate to the reservoir developed to date. The reservoir has significant storage capacity to attenuate large inflows. Therefore, because of this storage volume, there is no need for small capacity pumps to maintain a target operational stage within the reservoir. Therefore a basic assumption that needs to be made at this point is there is no apparent reason for the station to have pumps of different capacities. This is a significant positive aspect in regard to construction and installation costs requiring identical intakes, operating floor equipment layout, and equipment and auxiliary installation. Having identical drivers, reducers, and pumps reduce the spare part inventory, operator training, and the necessary operator experience, making the station significantly simpler to operate.

Given this simplification of equal pump capacities, it now becomes a decision of the number of the pumps. There are two cost items of the station that have a major influence on the station's total cost and represent a good indicator of the least cost alternative, the pump intake, and the pump equipment and its auxiliaries. The incremental cost of equipment does not change in a linear manner with decreasing capacity. In fact an engine model may be applicable to two alternatives since manufacturers produce engine models to address a range of horsepower requirements. This is also true of the pump and reduction gears. Therefore going from (4) pumps to (5) may not change the model of the equipment, only the design of some of its components, i.e. shaft size and propeller geometry. This is true with the auxiliary systems as well where there may be even less selection in service ratings for this support equipment, i.e. service water cooling and lubrication systems. The small incremental reduction in equipment cost is more than off-set by the increase in construction and labor costs. The more systems to install, the labor cost increases almost linearly. The time to install a 150 gallon day tank vs. a 125 gallon day tank, or a 1 inch diameter fuel line vs. a 1-1/2 inch line is more or less the same. Therefore the more pump equipment systems and their auxiliaries to be supplied and installed the greater the total cost of this item of work. The axiom in regard to equipment, the fewer the number of pumps the lower the cost.

For this TM, one objective is to determine the optimum size of the station's pump capacities based on a life cycle cost analysis. Since the reservoir's future operational scenarios have yet to be determined, it was considered prudent to include and array of pump numbers from the traditional optimum number of (3) to (5). Therefore, alternative station designs with the capacity grouping of (3) 1000 cfs, (4) 750 cfs, and (5) 600 cfs were selected for the analysis. The analysis shall therefore provide useful information for the future design given the need for a smaller incremental inflow rate to the reservoir.

3.3.5 Pump Alternatives

With the determination of the number of pumps and their respective rated capacities, the designer can select a pump type that satisfy the basic flow and head requirements. The head conditions for the proposed station alternatives are in the low head range. Axial flow pumps deliver large capacity flows at low heads and have specific speed in the range of 10,000 to 15,000+.

$$Ns = Nt\,(Q^{0.5})/H^{0.75}$$

Ns = pump specific speed
Nt = pump rotative speed, (rpm)
Q = flow at the BEP, (gpm)
H = head at the BEP, (feet)

This pump type also maintains good efficiency at a constant speed over a considerable greater range of heads compared to the lower specific speed pumps and is particularly applicable to variable head duties. The suction lift of this type pump is negligible. These pumps are typically used for flood control, water supply, or drainage applications where low head conditions exist and high capacity is required. They are capable of low heads from 6 to 20 ft. with capacities up to 500,000 gpm, (1110 cfs).

The advantages of axial flow pumps include:

- Ability to address large flow requirements.
- Relatively inexpensive.
- The pump can be easily changed by changing an impeller, to satisfy a new duty point.
- Pull out designs allow the rotating element be quickly and easily removed for inspection. For long pumps the pull out design reduces crane capacity requirements.

The disadvantages:

- Not capable of heads over 20 feet.
- No suction lift.
- Slow rotative speed of the pump requires a gear transmission to reduce the shaft speed of the driver or use of a slow speed pump.
- Water lubricated bearings result in higher maintenance requirements.

The low head conditions of the system designs of all alternatives of the pump station will require axial flow pumps. Vertical pumps are the most common style pump currently being used in pumping stations. Typically the slow rotative speed of the pump requires a reduction gear transmission to reduce the shaft speed of the diesel driver. Therefore the general arrangement is a vertical axial flow pump driven by a diesel engine through a right angle speed reducer. For smaller horsepower irrigation and drainage applications, horsepower requirements under 150 Hp, the axial flow pump is typically driven by a v-belt transmission and an electric motor. Vertical axial flow pumps have their impeller submerged and therefore are self-priming. For large pump units with an “over the levee discharge arrangement” and a siphon assisted delivery, vacuum systems are required to minimize the engine horsepower for start-up. Prior too removal of all air from the piping system, the pump must provide the head to raise the water up and over the siphon crest at a minimum flow and critical depth. For small to medium pump capacities the additional head generally encountered for start up conditions and the size of the pumping units do not justify the cost of a vacuum system for reduction in the driver's required horsepower. Therefore the pump and driver are selected for the maximum static head conditions for start-up. The additional expense and risk for automation or remote operation of the system is further justification for a self-priming pump.

For vertical style pump applications, the discharge pipe may be above or below the base plate, given there is a base plate. The below-base plate discharge style pump is the much more frequently used arrangement. The above the base plate discharge results in the driver at an elevation far above the operating level at the base plate and an elevated pump house floor. Site conditions may result in this arrangement being the best option. It does have an advantage in regard to the ease of disconnecting the pump from the discharge pipe. However, the above the base plate discharge is typically not used in the low head conditions of flood control pumping in South Florida because of the increase in head that is the result of the higher discharge pipe invert elevation.

Given no site constraints, it is often advisable to investigate the use of horizontal axial flow pumps for possible reduction of the height of the operating floor, reduction of the pump house square footage and the possibility of less system losses. Horizontal pumps have been utilized in many large capacity stations, some with pump propeller diameters greater than 120 inches. Some designers and operators prefer this style pump for the large capacity applications because of the better maintenance access to the drive shaft, bearings, and propeller. Horizontal pumps will require a vacuum priming system since the propeller is typically not submerged. They will also require a parallel shaft style reduction gear with the shaft in a horizontal position.

Rotodynamic pumps achieve their best efficiency at only one rate of flow and head, the best efficiency point (BEP). Pumps can operate satisfactorily within a hydraulic range to the left (low flow) and to the right (high flow) of BEP. For large stations with frequent usage, higher pump efficiencies can lower the installed horsepower requirements and reduce capital and operating costs significantly. Therefore, the station designer should consider higher efficiency equipment on a life cycle basis. Energy consumption can be reduced if the following factors are considered when specifying the pump:

- It may be desirable to select a pump that has a lower peak efficiency but a flatter efficiency curve to address the static head range.
- Engine: The efficiency of an engine is affected by the load. An engine will typically have a greater fuel consumption rate under part load.
- An objective of the pump selection and the station design is to achieve the lowest total head and highest efficiency to provide the smallest driver and the lowest energy cost.

Pumps have a preferred operating region, (POR), for head and capacity. Operation of the pump within this region will not significantly affect the service life of the pump by the additional hydraulic loading, vibration, and flow separation. The POR for most centrifugal pumps is between 70% and 120% of the BEP. For high specific speed pumps the POR is between 80% and 115% of BEP. The allowable operating region, (AOR), is the range of rates of flow and head over which the service life of the pump is not seriously compromised. Vibration levels exceeding the allowable limits is one criteria used by the pump manufacturers in establishing the AOR. Net positive suction head available (NPSHA), may also limit the AOR when the pump operation is over a wide range of flows.

There are a number of additional pump design considerations that become a concern with operation at the limits of the AOR including:

Flow velocities in the casing throat

- Stress limits of the shaft material
- Shaft fatigue
- Horsepower limitations
- Upthrust
- Suction recirculation
- Temperature rise
- Flow separation

The objective of the station designer as well as the pump designer is to ensure the POR satisfies the range of system heads. Operation outside this region should be limited. This objective is not just from the perspective of the service life of the mechanical components but also in regard to efficiency. Energy costs for pumps that see continuous use become the dominant factor in the LCC analysis. Operation left or right of BEP can result in significantly lower efficiencies and therefore much higher energy use. A pump is normally selected on the basis of a duty point, i.e. a required head and flow or a working range of flow and head conditions. The pump designer selects a pump model and speed that satisfies the system requirements and establishes the BEP.

There is a general relationship between specific speed and efficiency. The effect of pump size (capacity) is also significant.

Figure 1 illustrates that it can be concluded that for a high capacity application, the lower the specific speed of the pump the better opportunity for better efficiency. However, for a given duty point, a higher specific speed selection is a smaller pump at a higher speed, saving in both pump and driver cost. These factors must be weighed for the most cost effective and reliable design. The efficiencies of the figure represent the pump bowl efficiency which excludes losses within the pump components. The pump efficiency, the ratio of the pump output power to the pump input power that is the ratio of the water horsepower to the brake horsepower is somewhat less. Published data on pump efficiencies is not readily available from many manufacturers. Therefore bowl efficiencies shall be used in the analysis. In addition, since it is too premature to determine accurate efficiency maximums and minimums for the full operating range of the pumps, the efficiency at the rated condition shall be used in the energy calculations of the analysis.

The overall efficiency, the ratio of the energy imparted to the liquid by the pump to the energy supplied to the driver; that is the ratio of the water horsepower to the power input to the primary driver was estimated using 78% efficiency for the engine driver through the reduction gear. See the latter discussion in the LCC analysis which attempts to estimate part load fuel consumption rate for the engine models selected for this analysis.

3.3.6 Pump Equipment General Arrangement

With the pump type established, station concept design alternatives can be developed that include appropriate intake and discharge structures that satisfy the hydraulic requirements as well as address the constraints and challenges of the site. To help with the development of the station's mechanical arrangement it is important to establish the operation and maintenance objectives including:

- Reliable facility capable of operation during extreme storm events to maintain flood protection.
- Efficient operation to minimize energy costs.
- Robust structural and mechanical systems to provide a long service life.
- Flexibility to adjust operation to possible future changed hydraulic conditions.
- Automated operation to reduce operating costs.
- Minimize maintenance costs by use of appropriate corrosive resistant materials and coating systems and the proper site and building design.

The selected concept designs are typically compared using a life cycle cost, (LCC) analysis to establish the best cost alternative. With selection of the station design and its mechanical arrangement the requirements and characteristics of the system can be detailed and a final selection of the pump model can be selected. Iterative modifications of the station and/or system design may be required to achieve an optimum design. Such changes may be as dramatic as the number of pumps or pump type such as double suction vs. single suction to better match horsepower requirements to the available drivers.

3.3.7 Intake Alternatives

The intake's primary function is to provide the required flow to the pump. This flow should be uniform with a minimum of rotational wakes, free of harmful debris, have enough depth to prevent the formation of free surface vortices, and have an approach speed that avoids flow separation at boundaries. The pump intake also performs a number of other functions including providing the foundation for the service bridge and the pump house. The design objective from a cost perspective is to keep the depth and footprint size of this substructure to the minimum necessary to satisfy the hydraulic criteria. Therefore the structural design for the operating platform functions should accommodate the intake's minimum hydraulic geometric design in lieu of enlarging the intake to address these activities. However, in the case of rectangular intakes, from a hydraulic perspective, longer is better. So, if there is a need to increase the size of the intake, increasing its length will improve the flow characteristics to the pump.

The intake design is typically the responsibility of the pump purchaser. There are only a few available guidelines for the design of the pump intake, the more notable are the Hydraulic Institute Standard ANSI/HI 9.8-1998-Pump Intake Design, the British Hydraulics Research Association (BHRA) publication by Prosser, 1977 , U.S. Army Corps of Engineers Technical Letter Report by Triplett -1988 and the design guide recommended by Ingersoll-Dresser Pump Company -1991. The Corps of Engineers Engineering Manual EM 1110-2-3105 also provides sump design guidance. The standard generally used in North America to design the geometry of the intake is the ANSI/HI 9.8-1998 - Pump Intake Design Standard. In accordance with section 9.8 of the HI standard, each pump shall be installed in its own bay. The pump bay shall have adequate width and depth to limit the approach velocity to 1.5 ft./sec.

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The intake structure should be designed to allow the pumps to achieve their optimum hydraulic performance for all operating conditions. A good design ensures that the adverse flow phenomena are within the limits outlined in ANSI/HI Section 9.8.5.6. The hydraulic conditions that can adversely affect the pump performance and should not be present to an excessive degree are:

- Submerged vortices: Submerged vortices cause rapid changes in the local pressure on the pump propeller as a vortex core is ingested. This will result in severe vibration and cavitation.
- Free-surface vortices: Similarly, free surface vortices will cause rapid changes in the local pressure on the pump propeller as the vortex core is ingested, resulting in reduction of the pump discharge and a loss of efficiency. This, and any other air ingestion can cause fluctuations of impeller load which result in noise and vibration which may lead to physical damage.
- Swirl of flow entering the pump: Swirl is the ratio of the rotational and axial velocity components in the pump column. Swirl exists in the flow entering the pump if the tangential component of velocity is present in addition to the axial component. This condition does not comply with design assumptions in regard to the inlet velocity vector at the propeller vanes. Therefore, swirl in the pump intake can cause a significant change in the operating conditions for a pump resulting in changes in the flow capacity, power requirements and efficiency. It can also result in local vortex-type pressure reductions that induce air cores extending into the pump. Severe swirling flow or pre-rotation, when centered on the pump axis has an additional effect on the pump performance that can either enhance or reduce the pump's performance depending on the direction of the rotation.
- Non-uniform spatial distribution of velocity at the impeller eye: Unsteady flow causes the load on the impeller to fluctuate, which can lead to noise, vibration and bearing problems.
- Excessive variations in velocity and swirl with time
- Entrained air or gas bubbles: Air gulping or aerated flow will reduce pump discharge and loss of efficiency. Small quantities of air can result in significant efficiency drop, i.e. 3% free air showed a drop of 15% efficiency.

Ideally, the flow of water into any pump should be uniform, steady, and free from swirl and entrained air. Lack of uniformity can cause the pump to operate away from the optimum design condition, and at a lower hydraulic efficiency. The negative impact of each of these phenomena on pump performance depends on pump specific speed and size, as well as other design features of the pump that are specific to a given pump manufacturer. In general, large axial flow pumps (high specific speed) are more sensitive to adverse flow phenomena than small pumps or radial flow pumps (low specific speed). A more quantitative assessment of which pump types may be expected to withstand a given level of adverse phenomena with no ill effects has not been performed.

In designing an intake structure, the following points must be considered:

- Flow from the forebay should be directed toward the pump inlets in such a way that the flow
  reaches the inlets with a minimum of swirl.
- In order to prevent the formation of air-entraining surface vortices in the sump, the walls must be designed to avoid stagnation regions in the flow. A properly placed wall close to the inlet can reduce the tendency toward localized swirl and vorticity. The liquid depth also must be great enough to suppress surface vortices.
- Although excessive turbulence or large eddies should be avoided, some turbulence does help to prevent the formation and growth of vortices.
- The sump should be as small and as simple as feasible to minimize construction costs. However, the required sump volume may be specified for other reasons, such as to provide for a minimum or maximum retention time.
- Large scale turbulence within the intake will cause uneven blade loading resulting in pump vibration and noise.
- A distorted velocity profile caused for example by a clogged trash rack will promote swirl and vortex formation.
- Boundaries between stagnant regions and the main flow tend to be unstable and fluctuate on position. These regions promote unsteadiness in the main flow and increase the chances of the formation of air entraining vortices.

For flood control and water supply stations of large capacity there are two basic intake types:

- Rectangular Intakes
- Formed Suction Intakes

Rectangular intakes are the most common intake with the water entering from the forebay of the structure into the intake and then the bell of the pump. The intake dimensions are determined as multiples of the pump bell diameter. The inlet bell diameter is based on limiting the bell inlet velocity to 5.5 ft./sec.

Bell Area (ft^2) A = Pump Capacity Q / 5.5 ft./sec.

Bell Diameter (ft.) D = 2 (A / 3.14)^ 0.5

In accordance with the HI standard Section 9.7, the required minimum submergence to ensure there is no formation of free surface vortices is determined by the following empirical formula:

Submergence S = D + 0.58Q / D^1.5

The intake geometry recommended by the HI standard is based years of observation of constructed stations and numerous model studies. Table 2 summarizes the intake dimensions in accordance with the HI standards:

Formed Suction Intake (FSI) designs were developed by the US Army Corps of Engineers by the Hydraulics Laboratory in Vicksburg. The intake was developed to minimize the submergence needed as well as the width in comparison to the rectangular intake. The Corps experimented with a number of intakes and evaluated their performance based on the velocity distribution at the impeller. The geometry for the intake is presented in the Corps ETL 1110-2-327 as well as in Appendix I of EM 1110-2-3105. HI 9.8 also provides the geometry of the type 10 intake. The geometry is presented in terms of the throat diameter which commonly taken as the impeller diameter. The use of a FSI is typically limited to the large capacity pumps, 84 in. diameter and larger.

There is a significant difference in the submergence as determined by the Corps EM 1110-2-3105 and HI 9.8 with the HI intake being considerably deeper. Therefore the use of the Corps method will significantly reduce pump station excavation, dewatering and substructure costs. The use of the Corps method for determining submergence should be used with caution since there have been a number of physical model tests performed for various projects where the intake developed unfavorable flow conditions. Therefore for this project HI 9.8 shall be followed for the submergence calculations. During the physical model tests reduction in submergence shall be explored. At this point in the design, following Hydraulic Institute Standard 9.8 standards, there is very little difference between the calculated submergence of the two intake types. Subsequently, there are also only a few feet of difference in intake channel floor elevation between the two types of intakes. The FSI, however, can provide a significant difference in the width and length of the intake. Given the operating floor space requirements above do not govern the substructure size, (they typically do), there can being a footprint size reduction using the FSI.

It should also be noted, the ITT Industries has conducted model testing for their wet column mixed-flow pumps with both suction bell and the USCOE Type 10 FSI intakes. This testing was completed for the Davis Pond Diversion Project in St. Charles Parish, Louisiana. The rating for the prototype pump was 190 cfs at 16.6 TDH. The testing results revealed peak efficiencies with the FSI to be 81 percent; the same prototype tested with a suction bell intake configuration had approximately 87 percent peak efficiencies. ITT Industries attributes this difference to the FSI either due to additional head loss and/or because of a poorer flow pattern into the impeller.

The intake shall be provided a means to be dewatered for inspection, maintenance and/or removal of the pumps. For smaller stations with an intake height from floor to the bridge of between 14 and 18± ft. the intake can be designed to be dewatered in a similar manner as the District’s spillways, with a needle beam and dewatering needles. The standard dewatering needles are aluminum interlocking structural frames with a height of 22 ft. and widths of 2, 3, and 4 ft. The bottom of the needles are placed on a sill provided in the base slab and are leaned against the needle beam at a 10:1 angle. The needle beam may be a permanent installation or may be placed in vertical slots in the abutments or piers at the time of dewatering. For larger stations with deep intakes, dewatering bulkheads are used. These are structural steel gate like bulkheads that are lowered into slots in the abutments and piers by a crane. The bulkheads span the width of the bay and have equal heights so when stacked the top elevation is above the maximum suction design stage.

3.3.8 Discharge Works Alternatives

There are a number of discharge arrangement variations which can generally be categorized into two different styles: “over the levee” or “through the levee.”

3.3.8.1 Over-the-Levee Discharge

For this discharge arrangement the invert of the discharge line or tunnel shall be equal to or be above the maximum discharge pool elevation to prevent backflow through the pumps when they are not running. This arrangement often includes the use of a siphon which can be justified on the energy saved due to the lower head when primed. The discharge line requires a relief/vacuum valve at the highest point to allow the venting of the air and closure when the siphon is established. The pump should be selected to operate over the entire range of heads provided by the siphon.

3.3.8.2 Through-the-Levee Discharge

For this arrangement the discharge line runs horizontally through the levee typically at an invert elevation that is established by the vertical height requirements of the pump. If the crest of the discharge pipe in always below the discharge pool elevation, this alternative will have no disadvantage in regard to energy usage relative to that of a siphon assisted discharge. However, this style discharge does require a means to prevent backflow through the pump when shut off. USACE EM 1110-2-3105 Mechanical and Electrical Design of Pumping Stations requires two means to prevent backflow. Typically the primary method for smaller discharge pipes is the use of a flap valve. For larger diameter pipes and discharge tunnels a motor operated slide or roller gate is specified. The second means of backflow prevention is a matter of interpretation of the Corps manual. Dewatering bulkheads are generally considered as an adequate backup for primary backflow prevention method. However, the Corps preference is to provide a second gate. The alternatives of this analysis, with the “through the levee” discharge arrangements and flap gates, (Alternatives No. 3 and 4), have a second gate. This gate, however, is located in the intake and provides a means for dewatering of the pump suction bay of inspection and/or removal.

The top lip or crest elevation of the exit opening of the discharge tunnel or pipe for those alternatives with a vacuum priming system and assisted siphon flow delivery will require a minimum submergence below the predicted low groundwater elevation of the discharge channel. This low water elevation was assumed to be +5.6 ft. NAVD. The top lip or crest was conservatively set well below this low water stage to ensure a vacuum in the tunnel/pipe can be obtained as well as ensure the development of the siphon.

3.4 Description of Alternatives

As discussed, there is some degree of uncertainty in regard operational criteria of the station. The historic hydrologic flows would indicate the need for possibly small capacity (less than 1000 cfs) flow capabilities. However, the future build-out of the EAA Reservoir/STA-3/4 system has yet to be determined. Therefore, the 3000 cfs station capacity shall be broken into an array of incremental capacities, (3)1000 cfs, (4) 750 cfs, and (5) 600 cfs pumps. In addition, in accordance with the recommendations of the District’s “Major Pumping Station Engineering Guidelines”, a horizontal pump arrangement shall be included for comparison to the vertical pump alternative.

In regard to intakes, a FSI was selected for the larger pump alternatives, Alternatives Nos.1, 2, and 3. The FSI will provide significant substructure width reduction in comparison with a rectangular intake while satisfying the operating floor space requirements, i.e. operating distance between engine drivers. The station substructure length is largely governed by the operating floor requirements, namely the service bridge, equipment lay down floor area within the pump house, control room, driver and reduction gear, and the trash rack and collection system. For the (5) pump alternative, Alternative No. 4, the rectangular intake's greater width affords additional space between the engine units, while the FSI begins to make the floor a bit too congested. Including the rectangular intake in the analysis also affords an opportunity to compare construction costs, admittedly not an apple to apples comparison, but possibly an indicator of the advantage of the rectangular intake's more inexpensive reinforced concrete unit costs.

The horizontal pump arrangement, obviously results in a number of uniquely different operating floor requirements. With the pump unit outside the pump house the area and height requirements of this facility are significantly reduced. However, the location of the control room must be located on one side of the house or the other. Since all the control rooms are elevated well above the operating floor level, location and orientation of the engine control panels to be within site of the operator should not be a problem. A standard Corps type 10 FSI was used for the horizontal pump. The District's "Major Pump Station Engineering Guidelines" includes a sectional view of a very long formed intake with no defined geometry provided. This intake is apparently the propietary design of the authors of the guideline so it was decided to use the Corp standard FSI. This results in an elbow prior to the pump. Typically a distance of (5) diameters from an elbow to the impeller is desired by the manufacturer to ensure a uniform flow to the impeller. This 50 ft. length was considered excessive, 30 ft. was provided, with the thought that flow straightening vanes could be provided in the elbow to help provide a uniform flow pattern.

Alternative No.1 utilized a similar "up and over the levee" discharge arrangement as G370 with a discharge tunnel that gradually increased in cross sectional area and exits below low water so a siphon assisted delivery is ensured. The discharge tunnel design followed the District's "Major Pump Station Engineering Guidelines" and the "Hydraulic Design Criteria" for the inflow pumping stations of Stormwater Treatment area 3/4, prepared by Burns & McDonnell, dated April 2000 in regard to target velocities, namely 6 fps at the tunnel crest and between 2 to 4 (3.3) fps at the discharge opening. The top lip of the tunnel opening was set well below minimum low groundwater stage. The width of the tunnel discharge opening is set at the same width as the intake bay.

Alternative No. 2 with the (3) 1000 cfs horizontal pump utilized a "through the levee" discharge arrangement. However, because of the large diameter of the pump, the pipe crest is above the maximum pool elevation. Therefore a discharge tunnel was added similar to Alternative No. 1, with its opening below low groundwater stage to again permit a siphon assisted delivery.

The last two alternatives, Alternatives No. 3 and 4 utilized a "through the levee" arrangement with a straight horizontal discharge and backflow gates.

Therefore, (4) alternatives were selected for the proposed 3000 cfs (1,346,499 gpm) EAA Reservoir inflow pump station. The following is a brief descriptive summary of each alternative.

3.4.1 Alternative No. 1

This alternative shall have (3) vertical axial flow pumps with a rated capacity of 1000 cfs (448,833 gpm) each. The intake shall be a formed suction intake (FSI) in accordance with the geometry defined in the Corps of Engineers Engineering Manual EM 1110-2-3105. The submergence requirements shall be in accordance with the Hydraulic Institute (HI) HI-9.8-1998, Pump Intake Design standard. The discharge arrangement shall be an "up and over the protection elevation" arrangement with the discharge being pumped over a weir crest to a discharge tunnel. The exit of discharge tunnel shall be submerged to permit siphon assisted delivery of the flow as well as partial recovery of the velocity head. A vacuum system shall be provided to assist in the development of the siphon and removal of air from the discharge tunnel. The pump shall be driven by a diesel engine driver through a right angle reduction gear.

3.4.2 Alternative No. 2

This alternative shall have (3) horizontal axial flow pumps with a rated capacity of 1000 cfs (448,833 gpm) each. The intake shall be a formed suction intake (FSI) in accordance with the geometry defined in the Corps of Engineers Engineering Manual EM 1110-2-3105. The submergence requirements shall be in accordance with the Hydraulic Institute (HI) HI-9.8-1998, Pump Intake Design standard. The discharge arrangement shall be a "through the levee" arrangement with the discharge flowing horizontally from the pump to a discharge tunnel. Gates shall required to prevent backflow. The exit of discharge tunnel shall be submerged to permit siphon assisted delivery of the flow as well as partial recovery of the velocity head. A vacuum system shall be provided to prime the horizontal pump. The pump shall be driven by a diesel engine driver through a parallel shaft reduction gear.

3.4.3 Alternative No. 3

This alternative shall have (4) vertical axial flow pumps with a rated capacity of 750 cfs (336,625 gpm) each. The intake shall be a formed suction intake (FSI) in accordance with the geometry defined in the Corps of Engineers Engineering Manual EM 1110-2-3105. The submergence requirements shall be in accordance with the Hydraulic Institute (HI) HI-9.8-1998, Pump Intake Design standard. The discharge arrangement shall be an "through the levee" arrangement with the discharge being pumped horizontally to the reservoir with use of gates to prevent backflow. The pump shall be driven by a diesel engine driver through a right angle reduction gear. *(This alternative was later found to be the least energy efficient and was not reviewed further.)*

3.4.4 Alternative No. 3A

After development of the system losses it was determined Alternative 3 be revised to include a turned down discharge and use of a siphon assisted flow delivery to make it more competitive with the other alternatives in regard to energy usage. This alternative shall have (4) vertical axial flow pumps with a rated capacity of 750 cfs (336,625 gpm) each. The intake shall be a formed suction intake (FSI) in accordance with the geometry defined in the Corps of Engineers Engineering Manual EM 1110-2-3105. The submergence requirements shall be in accordance with the Hydraulic Institute (HI) HI-9.8-1998, Pump Intake Design standard. The discharge arrangement shall be an "through the levee" arrangement with the discharge turned down with its exit submerged to take advantage of a siphon assisted delivery. Auto controlled slide gates shall be used to prevent backflow. The pump shall be driven by a diesel engine driver through a right angle reduction gear.

3.4.5 Alternative No. 4 and 4A

This alternative shall have (5) vertical axial flow pumps with a rated capacity of 600 cfs (269,300 gpm) each. The intake shall be rectangular intake in accordance with the geometry defined in the Hydraulic Institute (HI) ANSI/HI-9.8-1998, Pump Intake Design standard. The discharge arrangement shall be an "through the levee" arrangement with the discharge being pumped horizontally to the reservoir with use of gates to prevent backflow. The pump shall be driven by a diesel engine driver through a right angle reduction gear. *(Alternative 4A was developed to eliminate the flap gate of the original Alternative 4 design and replace it with a backflow electric operated slide gate to reduce energy losses. Alternative 4 was dropped from further review.)*

3.4.6 Intake Geometry for Proposed Alternatives

The formed suction intake geometry for the proposed alternative flow rate is shown in Table 3.

The rectangular intake geometry for the proposed alternative flow rate is shown in Table 4.

4. EVALUATION OF ALTERNATIVES

4.1 Structural Design Considerations

4.1.1 Foundation

The assumption was made that a mat foundation would provide a more than adequate foundation for the structure. The greatest bearing loads from the dead load of the structure typically occur during the construction period when the structure is dry. However, there is the analytic consideration that the lower substructure will actually be flooded prior to construction of the structure above. Nevertheless, a preliminary review indicates loading should be well below allowable soil bearing pressure.

4.1.1.1 Stability Analysis

The stability of the structure is not a concern but the selected alternative should be analyzed to ensure it is well within the safety factors required by the engineering design standards.

4.1.1.2 Reinforced Concrete Design

There was only a preliminary analysis performed to size of the various abutment walls, piers and wingwalls. All structure alternatives will provide ample opportunity for lateral support of the abutments. The other structural components, thicknesses, depths, etc. were based on experience with the intent to provide a conservation estimate of the concrete cubic yards that will be required to construct the major reinforced concrete components of the structure. The dimensional adjustments that will be required by the future detailed design of the structure will not dramatically change the initial cost of the alternatives.

4.1.1.3 Pump House

A typical precast concrete panel with steel frame construction was assumed for proposed pump house alternatives. The pump house shall be required to be designed to satisfy District wind load standards that will substantially increase the panel and resisting frame sizes over that required by the Florida Building code. Because of these wind load requirements precast double tees were used in the analysis. The future design stages should review the possible advantages of a metal roof joist and cast in place deck roof system. The building size varies in width with each alternatives from 50’ x 74’ to 50’ x 98’. The eave height varies with each alternative due to the clearance requirements for removal of the major equipment by the bridge crane. A 4 ft. parapet was used in the analysis as is recommended due to the wind load requirements. The bridge crane height and the resulting eave height shall need further verification when the equipment dimension are established.

4.1.2 Construction Considerations

Dewatering of the structure site to permit construction of the intake in the dry is a key construction task which will determine the success of the project. It was assumed a cofferdam will be required to dewater the site due to the relatively porous soil characteristics. The cofferdam tip elevations and top elevations were all assumptions that were applied equally to all alternatives therefore providing no advantage to any one alternative in this comparative analysis. It was assumed there would not be the need for tremie seal and the cofferdam could be dewatered from sumps by pumping. However, all of these assumptions must be analyzed during the next phase of the design of the station. This dewatering facility is envisioned as a steel sheet pile cofferdam of a foot print size just large enough to allow construction of the substructure. Internal bracing with a tie-back system would be necessary for structural support of the sheets.

4.2 Life Cycle Cost Analysis

The primary objective of the conceptual development of EAA Reservoir A-1 Pump Station alternatives presented in the BODR is to compare the estimated life cycle costs (LCC). These costs shall include:

- Initial Cost: The initial cost represents the cost of the station’s construction, equipment and the associated auxiliary systems.

- Energy Cost: This cost represents the cost of fuel consumed for pumping during an average year in accordance with the operational model established for this study.

- Operating Cost: Operating cost is the labor cost for operation of the pump station.

- Maintenance Cost: Maintenance cost include routine and major maintenance as recommended by the manufacturer to ensure the design service life.

- Decommissioning: This is the cost for decommissioning and demolition of the facility at the end of its service life. For this analysis this station will continue to operate beyond the analysis period and therefore this cost will be considered $0.

4.2.1 Initial Cost

The estimate of construction costs utilized the unit costs of the recently commissioned G-370 and G-372 stations as well as available pricing from other recently constructed work. To provide a representative value for the completed station, a cost for construction of the immediate site work was included. It was considered important to include all the divisions of work in the initial cost estimate so that there would be no confusion as to the projected cost of the station. However, the cost of the approach canal to the North New River as well as the $4,000,000 ± bridge for Highway 27 were not included in this estimate. Also, there were no provisions for seepage control included in the alternative station design concepts. There are a number of divisions of work that have very little cost difference between the alternatives, namely the overhead, electrical, communications and control, and the HVAC. Other than Alternative 4A that has a notably wider intake and therefore greater forebay widths, the site work for the alternatives is identical. A 20% contingency was added to the initial construction cost estimate again to ensure the opinion of cost at this stage of the design development was conservative. See Appendix "D" for the breakdown of the estimated initial cost of the station alternatives. Table 5 is a summary of the costs:

4.2.2 Energy Cost

To summarize the annual station pumping operation:

Wet Season: 30.6 days (734.4 hrs) @ 2000 cfs with an average reservoir stage of 14.24 ft. NAVD or the total hours of pumping operation would be:

- Alternative no. 1 & 2, (2000cfs/1000cfs/pump) x (734.4 hrs) = 1470.8 pumping hrs.
- Alternative no. 3A, (2000cfs/750cfs/pump) x (734.4 hrs) = 1958.4 pumping hrs.
- Alternative no. 4A, (2000cfs/600cfs/pump) x (734.4 hrs) = 2448 pumping hrs.

Dry Season: 35.32 days (847.7 hrs) @ 3000 cfs with an average reservoir stage of 16.15 ft. NAVD or the total hours of pumping operation would be:

- Alternative no. 1 & 2, (3000cfs/1000cfs/pump) x (847.7 hrs) = 2543.1 pumping hrs.
- Alternative no. 3A, (3000cfs/750cfs/pump) x (847.7 hrs) = 3390.8 pumping hrs.
- Alternative no. 4A, (3000cfs/600cfs/pump) x (847.7 hrs) = 4238.5 pumping hrs.

An 80% pump efficiency and a 96% reduction gear efficiency for an overall efficiency of 78% for each alternative was utilized for the analysis for both the wet season and dry season operations. The average flow rate was taken as the rated pump capacity. To determine a representative fuel consumptive rate an engine model was selected for each alterative based on the rated pump conditions with consideration given the maximum horsepower requirements, see Table 6

Caterpillar engine models were used for the analysis. The following models were selected for the alternatives based on the above horsepower requirements:

Alternative 1: Caterpillar model 3606 w/ a maximum continuous rating of 2722 bhp and a continuous service rating of 2481 bhp at 1000 rpm. This is an in-line 6 cylinder, 4- stroke diesel engine. The fuel consumption rate is stated as 0.324 to 0.339 lb/hp-h, (7.001 lbs/US gal.).

Alternative 2: Caterpillar model 3606 w/ a maximum continuous rating of 2722 bhp and a continuous service rating of 2481 bhp at 1000 rpm. This is an in-line 6 cylinder, 4- stroke diesel engine. The fuel consumption rate is stated as 0.324 to 0.339 lb/hp-h, (7.001 lbs/US gal.).

Alternative 3A: Caterpillar model 3516B w/ a continuous service rating of 1853 bhp at 1200 rpm. This is a V-16 cylinder, 4- stroke diesel engine. The fuel consumption rate is stated as:

- 88.3 gph at 100% load.
- 65.2 gph at 75% load.
- 44.0 gph at 50% load.
- 24.7 gph at 25% load.

Alternative 4A: Caterpillar model 3512B w/ a continuous service rating of 1686 bhp at 1500 rpm. This is a V-12 cylinder, 4- stroke diesel engine. The fuel consumption rate is stated as:

- 77.1 gph at 100% load.
- 59.5 gph at 75% load.
- 41.8 gph at 50% load.
- 23.7 gph at 25% load.

Therefore using the annual operating hours based on the theoretical operational model an estimate of the annual energy usage was calculated for each alternative.

The energy cost was determined using the unit cost for diesel fuel at $2.25/gal. (density at 7.0 lb/gal). See Table 7.

4.2.3 Operating Cost

The operating costs were considered equal for all alternatives and included an estimated annual labor breakdown shown in Table 8.

4.2.4 Maintenance and Repair Cost

The maintenance costs were assumed to be $8.00/hour of pump operation. Recent quotes from vendors have been in the neighborhood of $7.00/hr. to $7.50/hr. for smaller stations. Given the number of auxiliaries and the size of the station, the $8.00/hr. was assumed for this analysis for all alternatives. See Table 9.

4.2.5 LCC Analysis Summary

The life cycle cost describes the total cost of providing, running, maintaining the station and associated equipment for the service life of the facility. The costs estimated above that comprise the total LCC need to be aggregated to allow a comparison of the alternatives. Financial factors assumed for the analysis include:

- Inflation rate at 3.5%
- Interest rate at 6.0%
- Discount rate = interest rate - inflation rate = 2.5%
- Review period = 25 yrs.
- Discount factor for 25 yrs. and 2.5% discount rate = 18.47
- Cp/Cn for 25 yrs. and 2.5% discount factor = 0.58

A (25) year analysis period was used primarily because this period is the expected service life of most of the mechanical and electrical equipment. It would be at this point in time the station would go through a major overhaul. It is difficult to estimate the scope and value of this repowering project. A number of the C&SFFC Project pump stations have gone through or are going through this repowering phase. The electrically systems are usually completely replaced during these projects but the pumps are in excellent condition requiring little refurbishment. The engines are replaced with four stroke turbo-charged engines that meet current clean air regulations. The fuel systems have gone through a number of overhauls as a result of changing environmental laws in regard to spill prevention. In some recent projects the stations have been automated and modified for remote operation. It also should be noted the more energy efficient alternative has the least LCC. The biggest concern with this analysis is the escalation of the energy costs. This factor adds significant emphasis on the energy component. Table 1 summarizes the LCC Costs

4.3 Discussion of Analysis

The initial investment costs of the alternatives are surprisingly close. Given the margin of error at this early stage in the design process it can assumed that all the alternatives are equally competitive in regard to their construction costs. The primary reason for this result, Alternatives 3A and 4A, the alternatives with the greater number of pumps, do not have a concrete formed discharge tunnel, an expensive substructure component. In addition, Alternative 4A also does not have a FSI another expensive feature. However, the savings here is somewhat lost due to the size of the rectangular intake. Given similar substructure designs the trend would be an increase in the number of pump would result in an increase in initial cost. Table 10 summarizes the costs of the major features of the alternatives:

The following is an assessment of the alternatives in regard to their initial costs:

Alternative No.1

Advantages:

- Smaller footprint and therefore lower cofferdam costs.
- Smaller pump house square footage.
- Least number of pump system auxiliary connections.
- Reduced substructure width due to FSI.
- Reduced substructure length due to FSI.

Disadvantages:

- Tall substructure due to “up and over” discharge arrangement.
- Tall pump house due to equipment removal clearance requirements.
- Expensive cost for FSI construction.
- Expensive cost for discharge tunnel construction.

Alternative No. 2

Advantages

- Smaller pump house square footage.
- Lower eave height of pump house due to pump external to house.
- Reduced height of substructure due to “through the levee” discharge arrangement.
- Smaller bridge crane due to pump external to pump house.
- Least number of pump system auxiliary connections.
- Reduced substructure width due to FSI.

Disadvantages

- High cost of construction for FSI
- High cost of construction for concrete formed discharge tunnel.
- Need for backflow gates.
- Higher pump costs.
- Largest footprint therefore highest cofferdam costs.
- Longer substructure length due to orientation of driver and general mechanical arrangement required by the horizontal pump.

Alternative No. 3A

Advantages

- Reduced height of substructure due to "through the levee" discharge arrangement.
- Reduced cost of substructure due to piped discharge.

Disadvantages

- High cost of construction for FSI
- Need for backflow gates.
- Higher mechanical costs because of more pumps and auxiliary connections.
- Higher pump house cost because of larger square footage.
- Longer substructure due to orientation of driver.

Alternative No. 4A

Advantages

- Reduced height of substructure due to "through the levee" discharge arrangement.
- Reduced cost of substructure due to piped discharge.
- Reduced cost of substructure due to rectangular intake.

Disadvantages

- High cost of dewatering due to large footprint.
- Need for backflow gates.
- Highest mechanical costs because of the most number of pumps and auxiliary connections.
- Highest pump house cost because of largest square footage.
- Longer substructure due to orientation of driver.

The impact of the energy costs on the LCC results is not surprising given the number of operating hours developed by the operational model and the review of the head losses of the various alternatives. Alternative 1 and 2 with the discharge tunnel that recovers some velocity head is part of the reason for their more efficient operation. There may be the need after further study by the pump designers to increase the impeller diameter of Alternative 2 due to NPSHA concerns. This will result is slower flow velocities that will further reduce its losses. The increase in initial cost is unlikely to impact the outcome of the LCC analysis. The following is an assessment of the alternatives in regard to energy costs:

Alternative No. 1

Advantages

- Recovery of velocity head and exit losses due to discharge tunnel.
- Lowest operating static heads due to siphon assisted delivery.

Disadvantages
- Minor losses of FSI.
- Minor losses at pump elbow and transition to discharge tunnel.
- High weir crest elevation an start-up horsepower requirements.

Alternative No. 2

Advantages
- Recovery of velocity head and exit losses due to discharge tunnel.
- Lowest operating static heads due to siphon assisted delivery.
- Lowest friction losses due slowest flow velocities due to largest impeller.

Disadvantages
- Minor losses of FSI.
- High pipe crest elevation requiring large capacity vacuum system for priming pump.

Alternative No. 3A

Advantages
- Lowest operating static heads due to siphon assisted delivery.

Disadvantages
- Minor losses of FSI.
- High pipe crest elevation requiring large capacity vacuum system for priming pump.
- No velocity head recovery.
- High exit losses.

Alternative No. 4A

Advantages
- Low entrance losses due to bell intake.
- Lowest pipe crest elevation requiring low start-up horsepower without vacuum priming.

Disadvantages
- No velocity head recovery.
- High exit losses.

The assessment of the alternatives in regard to the reliability concerns and operation:

Alternative No. 1

Advantages
- No backflow gate required.

Disadvantages
- Requires vacuum system to initial siphon assisted delivery and minimize start-up horsepower.
- Requires dewatering facilities to remove pump.
- Pump must be removed in sections.

Alternative No. 2

Advantages
- Horizontal split case pump easily dismantled for inspection and repair.
- No dewatering facility needed for pump repair or removal.

Disadvantages
- Requires vacuum system to prime pump as well as initiate siphon assisted delivery.
- Requires backflow gate.

Alternative No. 3A

Advantages
- More pumps of smaller capacity provide pumping flexibility.
- More pumps adds less impact to station operation when pump unit down.

Disadvantages
- Requires vacuum system to initiate siphon assisted delivery.
- Requires backflow gate.
- Requires dewatering facilities to remove pump.
- Requires pump be removed in sections.

Alternative No. 4A

Advantages
- Self -priming no vacuum system required.
- The most number of pumps with the smallest capacity provides the most pumping flexibility.
- With the most number of pumps, provides least impact to station operation when pump unit is off line.

Disadvantages
- Requires backflow gate.
- Requires dewatering facilities to remove pump.

Given the relative minor difference in the present value totals for the alternatives and the margin of error in the estimating of not only the construction costs but also the potential pump performance at this early stage of the design, the LCC analysis does not provide a conclusive result as to the optimum station alternative. But there is an important finding that is evident from the analysis. For this station, which could see many more hours of continuous duty than the typical District pumping station, energy costs are extremely important. This is especially true considering the cost of fuel is escalating at a rate beyond that of the 3.5% inflation rate used in the analysis. Therefore the station design needs to incorporate every efficiency measure that is available to minimize system losses and reduce fuel consumption regardless of the initial cost. This will result in the optimum station design from a life cycle cost perspective.

5. RECOMMENDED ALTERNATIVE

As a consequence of the uncertainty of the future operation of the station, flexibility of operation ranks next in importance to fuel economy in the station design. Therefore, at this point in time given the initial costs for all the alternatives are in essence equal, the five (5) smaller capacity pump alternative is our selected alternative. The general arrangement section drawing entitled “Alternative 5” is a suggested arrangement that makes use of the advantages of the various alternatives outlined above, and provides the greatest flexibility for operation of the current and future flood control and water supply system. This station design has the following:

- Self-priming with no vacuum system required reducing potential complications for remote operation of station.
- The most number of pumps with the smallest capacity provides the most pumping flexibility.
- With the most number of pumps, provides least impact to station operation when a pump unit is off line.
- Low entrance losses due to bell intake.
- Lowest pipe crest elevation requiring low start-up horsepower without vacuum priming.
- Recovery of velocity head and exit losses due to discharge tunnel.
- Steel fabricated discharge tunnel to reduce friction losses and lessen construction cost.
- Lowest operating static heads due to siphon assisted delivery.
- Low friction losses due slower flow velocities as a result of the use of larger impeller and slower rotative speed for this pump capacity.
- Reduced height of substructure due to “through the levee” discharge arrangement.
- Control room and break rooms at opposite side of pump house from engines permitting optimum viewing of operating floor equipment and engine control panels.
- Control room and break rooms at opposite side of pump house from engine and exhaust system reducing noise.
- Engines located close to exterior wall permitting intake ventilators to be located in close proximity to engines for optimum ventilation arrangement.
- Electric start engines eliminating need for large capacity compressed air system and potential complication for remote starting.
- Reduced cost of substructure due to rectangular intake.

6. GLOSSARY OF TERMS

The following definitions apply to this specification and generally comply with ANSI/HI 2.1 - 2000 and ANSI/HI 2.6.3 - 2000.

BEST EFFICIENCY POINT (BEP): The rate of flow at which the pump efficiency (ηp) is maximum.

BOWL ASSEMBLY EFFICIENCY (ηba): The efficiency obtained from the bowl assembly, excluding losses within the pump components:

$$\eta ba = \frac{\text{Pw (pump output power) + other pump component mechanical and hydraulic losses}}{\text{Pba (bowl assembly input power)}} \times 100$$

BOWL ASSEMBLY INPUT POWER (Pba): The bowl assembly input power is the horsepower delivered to the bowl assembly shaft. See HI Engineering Data Book, second edition for an estimate of the line-shaft horsepower losses.

DRIVER INPUT POWER (Pdrv): The electrical power input to the motor driver (kilowatts), or pump input power, (Pp), divided by the speed reducer efficiency.

HEAD (h): Head is the expression of the energy content of the water referred to a datum elevation. It is expressed in units of energy per unit weight of the water. The unit used in this specification shall be feet, (ft.) of water.

MINIMUM SUBMERGENCE (S): The minimum water height over the suction bell inlet (Reference ANSI/HI 9.8.7.3 – 1998.)

OVERALL EFFICIENCY (ηoa): This is the ratio of the energy imparted to the liquid, (Pw) by the pump to the energy supplied to the driver (Pdrv); that is the ratio of the water horsepower to the power input to the primary driver expressed in percent.

PREFERRED OPERATING REGION (POR): The preferred operating region is the region over which the pump's vibration, noise, and cavitation are within acceptable limits.

PUMP EFFICIENCY (ηp): The ratio of the pump output power (Pw) to the pump input power (Pp); that is, the ratio of the water horsepower to the brake horsepower expressed in percent.

PUMP INPUT POWER (Pp): The pump input power (brake horsepower), is the power needed to drive the complete pump rotating assembly including the propeller-bowl assembly input power, line shaft power loss, stuffing box loss and thrust bearing loss. With pumps that rely on the driver thrust bearing, the thrust bearing loss shall be added to the power delivered to the pump shaft.

PUMP OUTPUT POWER (Pw): The power (water horsepower), imparted to the liquid by the pump.

PUMP TOTAL DISCHARGE HEAD (hd): The pump total discharge head (hd) is the sum of the discharge gauge head (hgd) measured after the discharge elbow plus the velocity head (hvd) at the point of the gauge attachment plus the elevation head (Zd) from the discharge gauge centerline to the pump datum.

PUMP TOTAL HEAD (H – commonly called “Total Dynamic Head” or “TDH”): The pump total head, (H) is the difference between the pump total discharge head, (hd) and the total suction head, (hs). This is the measure of the net energy increase per unit weight of the liquid, imparted to the liquid by the pump.

RATED CONDITION POINT: Rated condition point applies to the rate of flow, pump total head, speed, NPSHR, efficiency, and pump input power as required by the Contract specifications. The rated condition is the point at which the pump manufacturer certifies the pump’s performance is within the acceptance criteria tolerances stated in this specification.

RATE OF FLOW (Q): The rate of flow, (capacity), of a pump is the total volume through-put per unit of time at the suction conditions. Units used in this specification shall be U.S. gallons per minute, (gpm).

SHUTOFF HEAD: The pump total head (H) when the pump operates at the rated speed and the pump is at zero flow.

SPEED (n): The number of revolutions of the shaft in a given unit of time. Speed is expressed as revolutions per minute, (rpm).

TOTAL SUCTION HEAD (hs), Open Suction: The total suction head (hs) at datum is the vertical distance in feet from free water surface to datum. The average velocity head of the flow in the intake shall be neglected.

TABLES

Table 1 LLC Analysis

	Alt. 1	Alt.2	Alt. 3A	Alt. 4A
Initial Investment	$29,310,000	$29,420,000	$29,023,000	$29,356,000
Sum of Annual Costs	$ 1,235,000	$ 1,183,000	$ 1,329,000	$ 1,331,000
PV LCC Energy	$17,072,000	$16,109,000	$18,599,000	$18,441,000
PV LCC Operations	$ 5,150,000	$ 5,151,000	$ 5,150,000	$ 5,151,000
PV LCC Maintenance	$ 593,000	$ 593,000	$ 790,000	$ 988,000
PV LCC Annual Costs	$22,815,000	$22,816,000	$24,540,000	$24,580,000
Present Value LCC	**$52,125,000**	**$51,331,000**	**$53,622,000**	**$53,994,000**

Table 2 Intake Dimensions per HI Standards

Description	**Recommended Dimension**
Bell centerline to entrance of intake and location of trash rack	5D
Bell centerline to backwall	0.75D
Bottom of bell to floor	0.52
Min. water depth above floor	S+0.5D
Bay width	2D

Table 3 Formed Suction Intake Geometry

	Alt. No. 1	**Alt. No. 2**	**Alt. No. 3A**
Capacity at rated condition (cfs/gpm)	1000/448,830	1000/448,830	750/336,623
Throat Diameter (inches/feet)	10/120	10/120	8.5/102
Submergence (inches/feet)	290/24.2	290/24/2	257/21.4
Pump centerline to entrance (feet)	33.0	33.0	28.05
FSI height (feet)	8.80	8.80	7.48
FSI width (feet)	23.10	23.10	19.64
Min. water depth above floor (feet)	28.57	28.57	25.15
*Low Water shut-off elev. (feet NAVD)	5.6	5.6	5.6
Intake slab elev. (feet NAVD)	-20.71	-20.71	-16.75

*assumes (1) foot head differential across trash rack due to blockage at design low water

Table 4 Rectangular Intake Geometry

	Alt. 4A
Capacity at rated condition (cfs/gpm)	600/269,298
Bell Diameter (inches/feet)	11.79/109
Submergence (inches/feet)	234/19.5
Bell centerline to entrance (feet)	58.9
Bell centerline to backwall (feet)	8.8
Bell inlet to floor (feet)	5.9
Bay width (feet)	23.6
Min. water depth above floor (feet)	25.41
*Low Water shut-off elev. (feet NAVD)	5.6
Intake slab elev. (feet NAVD)	-17.01

*assumes (1) foot head differential across trash rack due to blockage at design low water

Table 5 Pump Station Cost Estimate

Description of Cost	Alt. 1	Alt.2	Alt. 3A	Alt. 4A
Overhead	$1,808,000	$1,808,000	$1,808,000	$1,808,000
Dewatering	$1,235,800	$1,492,000	$1,297,000	$1,423,600
Structure	$7,005,800	$6,372,150	$5,920,700	$5,554,950
Mechanical	$9,789,000	$10,263,500	$10,384,100	$10,522,000
HVAC	$158,000	$158,000	$158,000	$158,000
Electrical	$1,202,500	$1,197,500	$1,202,000	$1,202,500
Communications/Control	$925,700	$925,700	$1,111,400	$1,297,100
Site Work	$2,255,100	$2,255,100	$2,255,100	$2,443,400
Subtotal	$24,255,100	$24,516,950	$24,186,300	$24,463,550
20% Contingency	4,884,980	$4,903,390	$4,837,260	$4,892,710
TOTAL Pump Station	**$29,309,880**	**$29,420,340**	**$29,023,560**	**$29,356,260**

Table 6 Rated Pump Conditions

Description	Alt. 1	Alt.2	Alt. 3A	Alt. 4A
Hp at rated condition	2221	2104	1836	1450
Max. Hp	2548*	2719*	1942*	1532
Wet Season Ave. Hp	1951	1834	1633	1275
Dry Season Ave. Hp	2228	2112	1841	1520
*with vacuum assist to initiate siphon				

Table 7 Summary of Energy Costs Used in the Analysis

Alternative 1						
Season	**Season Input Hp.**	**% Load**	**Fuel rate (gph)**	**Run hours**	**Fuel (gal)**	**Energy Costs [$/yr]**
Dry	2228	89	107.4	2543.1	273,129	$614,540
Wet	1951	78	93.6	1470.8	137,667	$309,750
Total				4013.9	410,796	$924,290
Alternative 2						
Dry	2112	85	101.6	2543.1	258,379	$581,352
Wet	1834	73	87.8	1470.8	129,253	$290,821
Total				4013.9	387,632	$872,173
Alternative 3A						
Dry	1841	99	87.4	3390.8	296,356	$666,801
/Wet	1633	88	77.2	1958.4	151,188	$340,174
Total				5349.2	447,544	$1,006,975
Alternative 4A						
Dry	1520	90	70.1	4238.5	297,712	$668,517
Wet	1275	75.6	59.9	2448	146,635	$329,929
Total				6686.5	444,347	$998,446

Table 8 Estimated Operating Costs

Job Description	**Rate/hr.**	**Est. hours**	**Total**
Mechanic	$55	700	$38,500
Mechanics Assistant	$45	700	$31,500
Operator	$55	1500	$82,500
Electrician	$60	500	$30,000
General Labor	$40	1500	$60,000
Subtotal			$242,500
Admin/Mgmt Support 15%			$36,375
Total			$278,875

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Table 9 Maintenance and Run Costs

Description	Alt. 1	Alt.2	Alt. 3A	Alt. 4A
Run Hours	4013.9	4013.9	5349.2	6686.5
Maintenance Costs	$32,111	$32,111	$42,794	$53,492

Table 10 Costs of the Major Features of the Alternatives

Design Component	Alt. 1	Alt.2	Alt. 3A	Alt. 4A
Dewatering	$1,235,000	$1,492,000	$1,297,000	$1,423,000
Substructure	$6,022,000	$5,518,000	$4,891,000	$4,299,000
Pump House	$ 983,000	$ 854,000	$1,029,000	$1,255,000
Mechanical	$9,789,000	$10,263,000	$10,384,000	$10,522,000

FIGURE

Figure 1 Efficiency-Specific Speed Relation